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The effect of cable loosening on seismic response of a prestressed concrete cable-stayed bridge

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Abstract

In conventional non-linear seismic analyses of cable-stayed bridges, the non-linear characteristics of the girders, stay cables and towers are considered. The non-linearity caused by cable loosening should also be considered because a large axial force fluctuation is generated in the cables of a prestressed concrete (PC) cable-stayed bridge that is subjected to strong seismic motion. In this paper, the possibility of the cable loosening in a PC cable-stayed bridge is discussed by using a cable model that can express the cable loosening. Furthermore, the effect of the cable loosening on the responses of the cables, girder and towers is evaluated using the mean value for three seismic waves. Numerical analytic results imply that the cable loosening appears in the bottom cables of the multi-cable system and the dynamic response of the bridge is slightly increased.

1. Introduction

Generally, the seismic design of a cable-stayed bridge takes into consideration the non-linear characteristics of the girders, stay cables and towers. Many researches have been reported in the literature about non-linear seismic response analysis. Fleming and Egeseli studied the 2-D geometric non-linear seismic responses of a cable-stayed bridge [1]. Nazmy and Abdel-Ghaffar studied the 3-D geometric non-linear seismic behavior of long cable-stayed bridges in the United States [2–4]. A tangent stiffness iterative procedure was used in the analysis to capture the non-linear seismic response. Wang and Yang performed parametric static studies on cable-stayed bridges for investigating the influence of sources of non-linearities, which include the large deflection, beam-column and cable sag effects [5].

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Non-linear seismic responses of bridges under strong earthquake motion were considered in Japan after the 1995 Great Hanshin Earthquake [6,7]. Ren and Obata studied the elastic–plastic seismic behavior of a long span steel cable-stayed bridge through a plane finite element model [8]. Both geometric and material non-linearities were involved in the analysis. The geometric non-linearities came from the stay cable sag effect, axial force–bending moment interaction, and large displacements, while material non-linearities arose when the steel girder yielded.

The effect of fluctuant axial force on non-linear seismic response of a prestressed concrete (PC) cable-stayed bridge, whose mass is greater than that of the steel cable-stayed bridge, was investigated at first in Japan by Aso et al. [9]. In this paper, it is assumed that non-linearity is affected by fluctuant axial force, i.e., crack and yield points of members vary due to the interrelationship between bending moment and axial force. However, the cable was considered to be a linear structure.

The non-linear characteristics of cables of a PC cable-stayed bridge should also be considered in an analysis because a large axial force fluctuation is generated in the cables of a PC cable-stayed bridge subjected to strong seismic motion, which induced large seismic forces. The non-linear cable model must include the cable loosening since the cable cannot resist compressive force as treated by Wang and Yang [5]. Therefore, earthquake response of a PC cable-stayed bridge should be examined by considering non-linear characteristics of the girder, towers and cables. Little research has been done on the non-linearity of cable loosening in cable-stayed bridges under seismic loading, while hangers loosening of suspension bridges is studied by many authors [10–12].

Kono and Kawashima examined the effects of cable loosening on the responses of a PC cable-stayed bridge using a 2-D model subjected to seismic motion in the longitudinal direction [13]. However, the girder was considered to be a linear structure and the initial dead load stresses were not considered.

This paper attempts to examine the possibility of cable loosening in a PC cable-stayed bridge. A 3-D bridge model is used, and the initial stresses are considered. A non-linear dynamic analysis of a PC cable-stayed bridge is carried out for ground motions in a single direction (longitudinal, lateral or vertical) or in a plane (both longitudinal and vertical directions). The girder and towers have a non-linearities prescribed by the relationship between moment and curvature, while stay cables have non-linearities due to sag effect and loosening.

Furthermore, in order to evaluate general tendencies, the effect of the cable loosening on the responses of the cables, girder and towers is evaluated by using the method of the mean values for three strong ground motions of the Great Hanshin Earthquake records, which is recommended in Ref. [6].

2. Method of analysis

2.1. Analytical model

The bridge analyzed in the research reported in the paper is a PC cable-stayed bridge located in Japan (Fig. 1). The bridge has three spans: a main span, 219.0 m in length, and two side spans, each 102.7 m in length. The deck cross-section is an aerodynamically shaped closed box PC girder

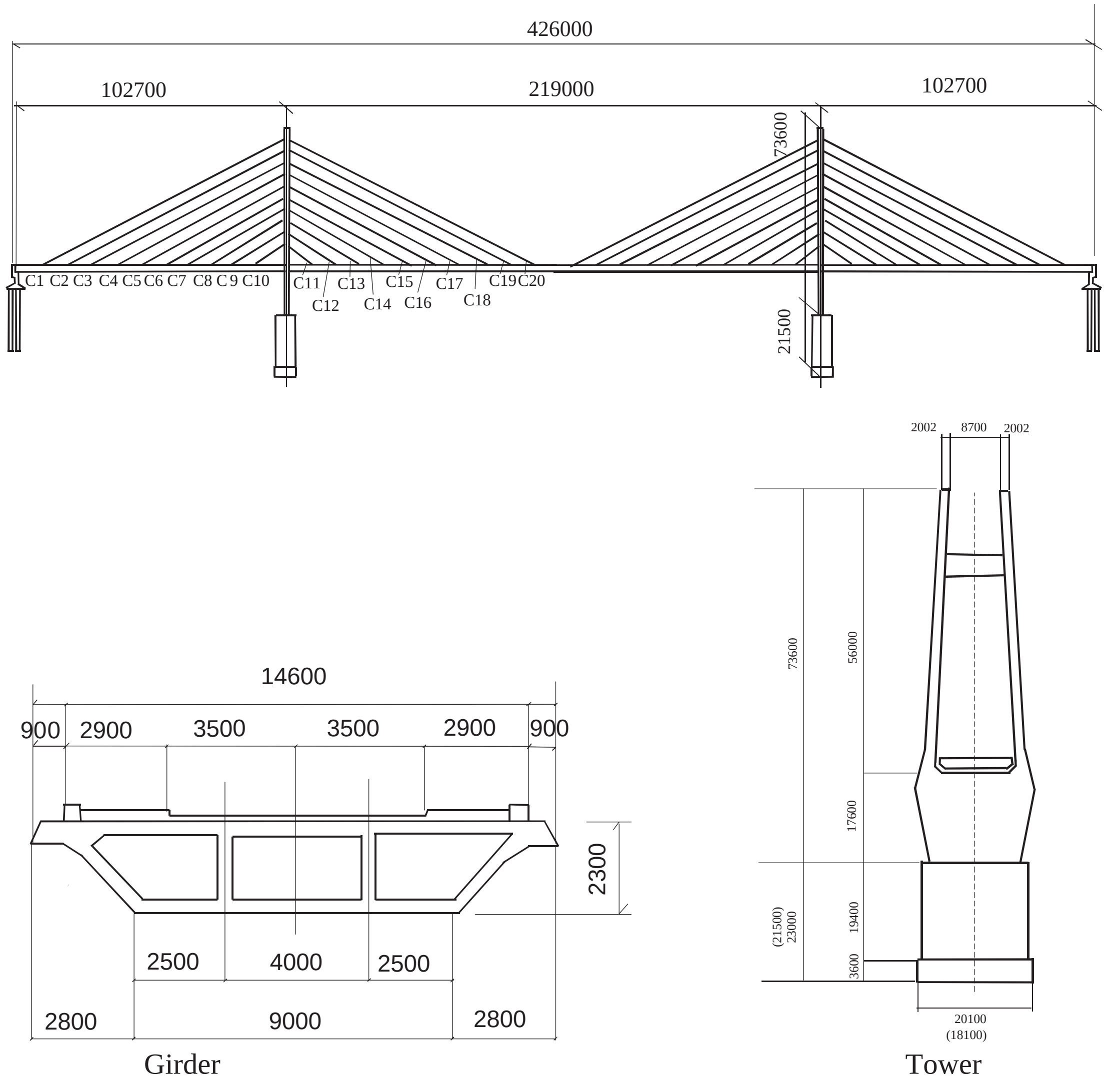


Fig. 1. General view of the cable-stayed bridge (unit: mm).

14.6 m in width and 2.3 m in height. The towers are H-shaped, and the cables are a two-plane, multiple-cable system arranged in a semi-harp pattern.

The cable numbers and the indicated nodes on the girder and towers are shown in Fig. 2. The cables are numbered sequentially from the side span to the main span.

2.2. Non-linear characteristics of girder and towers

For the 3-D FE model of the bridge, the girder is modelled using a single central beam with offset links to the cable anchor points. The towers are modelled by using 3-D beam elements based on cross-sectional properties. Regarding the boundary conditions, the girder is free to move

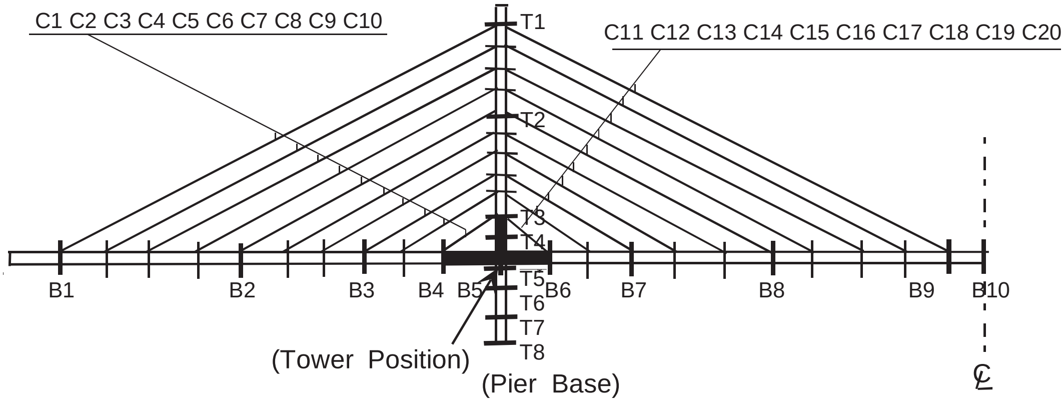


Fig. 2. Cable numbering and indicated nodes.

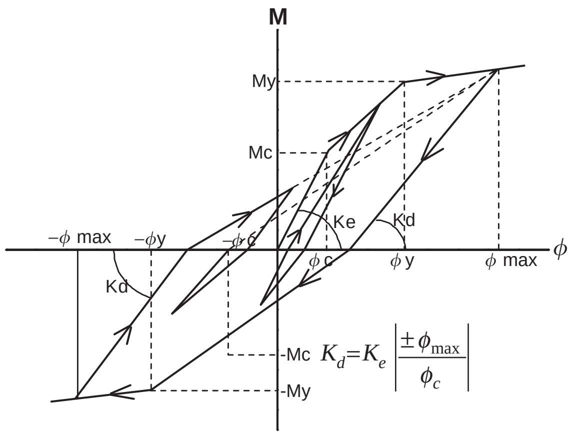


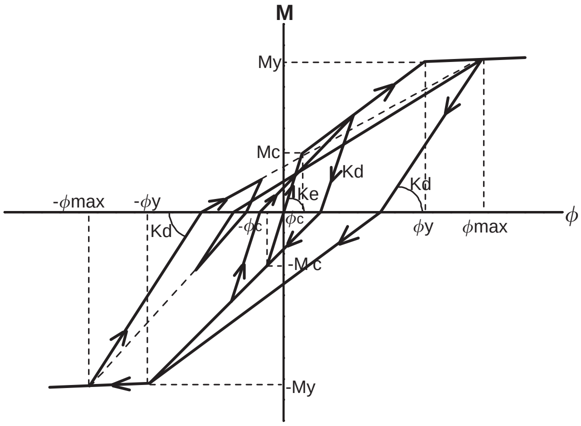
Fig. 3. Takeda hysteretic property.

in the longitudinal direction and is restrained in the vertical and lateral directions. The rotational component around the longitudinal axis is only restrained. The tower bases are fixed in all degrees of freedom at the ends.

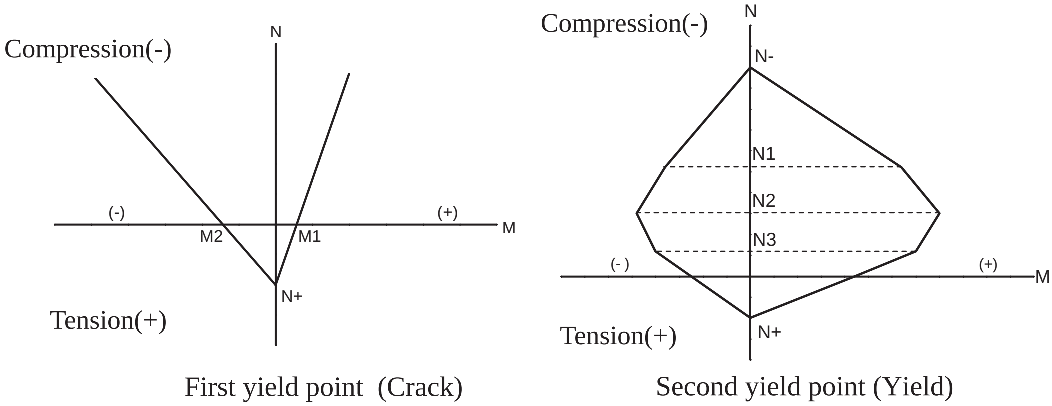
The girder is analyzed using a non-linear model that neglects axial force fluctuation. The Takeda Hysteretic model proposed by Takeda et al. [14] is adopted as restoring force characteristics ($M-\Phi$) shown in Fig. 3. The towers are analyzed using a non-linear model that considers axial force fluctuation. The Edo model is applied as shown in Fig. 4. When axial force fluctuation is considered, breakpoints of skeleton curve are recalculated by the relationship between axial force and bending moment ($N-M$) [15].

2.3. Non-linear model for stay cables

The cables are modelled as single non-linear springs with initial tensions and with no mass. The cable sag effect firstly suggested by Ernst [16] is considered in the present analysis. The relationship between the axial force and the axial displacement of the cables is shown in Fig. 5.



(a) Restoring force characteristic (M-Φ)



(b) M-N correlation

Fig. 4. Edo hysteretic model.

The axial displacement is measured from the no stress condition. The axial force and the stiffness of the cable become zero in the region of compressive force. The unloading curve is assumed to be the same as that of the skeleton curve.

In this paper, the non-linear cable model considers the effect of the cable loosening, while the linear cable model does not. Dimensions and forces of all cables are summarized in Table 1.

2.4. Numerical analysis method

The non-linear differential equations of motion are directly integrated. The Newmark β method ($\beta = 0.25$) is used. The time interval of numerical integration is 0.005 s.

Rayleigh damping is employed in the present analysis. The damping matrix takes the form

$$[C] = a_3[M] + a_4[K], \tag{1}$$

in which $[M]$ is the mass matrix, $[K]$ is the stiffness matrix, and a_3 and a_4 are arbitrary proportional factors. These factors are determined by assuming two damping constants of

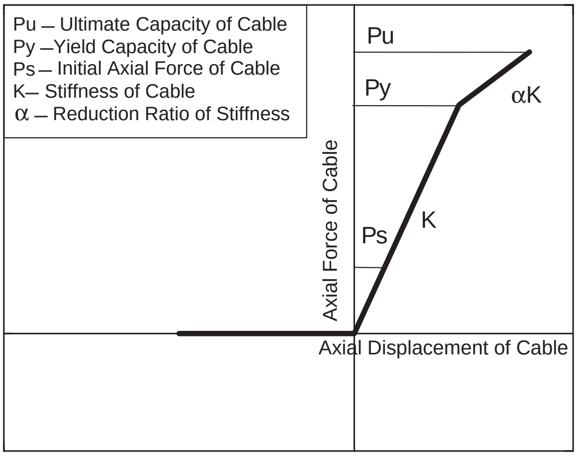


Fig. 5. Relationship between axial force and axial displacement of non-linear cables.

Table 1
Dimensions and forces of cables

	θ (°)	w (kN/m)	L (m)	A (m^2)	E (kN/m^2)	P_s (kN)	P_y (kN)	P_u (kN)
C1	61.060	0.996	108.230	0.006658	1.92×10^8	2860	10620	11750
C2	60.980	0.944	98.231	0.005825	1.93×10^8	2650	9290	10280
C3	60.862	0.704	88.259	0.005132	1.97×10^8	2440	8190	9070
C4	60.704	0.701	78.265	0.005132	1.98×10^8	2520	8190	9070
C5	60.491	0.697	68.308	0.005132	1.98×10^8	2530	8190	9070
C6	60.140	0.694	58.239	0.005132	1.99×10^8	2460	8190	9070
C7	59.618	0.689	48.189	0.005132	1.99×10^8	2300	8190	9070
C8	58.796	0.681	38.130	0.005132	1.99×10^8	2020	8190	9070
C9	57.332	0.667	28.121	0.005132	1.99×10^8	1630	8190	9070
C10	54.068	0.637	18.214	0.005132	1.99×10^8	1150	8190	9070
C11	55.110	0.713	17.965	0.005132	1.99×10^8	1220	8190	9070
C12	59.577	0.713	28.623	0.005132	1.99×10^8	1970	8190	9070
C13	61.487	0.713	39.391	0.005132	1.99×10^8	2440	8190	9070
C14	62.536	0.713	50.214	0.005132	1.99×10^8	2730	8190	9070
C15	63.193	0.713	61.057	0.005132	1.99×10^8	2870	8190	9070
C16	63.638	0.713	71.913	0.005132	1.99×10^8	2930	8190	9070
C17	63.956	0.713	82.784	0.005132	1.98×10^8	2870	8190	9070
C18	64.192	0.713	93.657	0.005132	1.97×10^8	2720	8190	9070
C19	64.372	0.953	104.550	0.005825	1.93×10^8	2820	9300	10300
C20	64.512	0.970	115.430	0.006103	1.89×10^8	2580	9740	10780

θ: inclined angle, w: weight per unit length, L: length, A: cross-sectional area, E: Young's modulus, P_s, P_y and P_u: initial axial force, yield capacity and ultimate capacity, respectively.

arbitrary two modes such as [17]

$$a_3 = \frac{2\omega_i\omega_j(h_i\omega_j - h_j\omega_i)}{\omega_j^2 - \omega_i^2}, \quad a_4 = \frac{2(h_j\omega_j - h_i\omega_i)}{\omega_j^2 - \omega_i^2}, \tag{2}$$

in which ω_i is the natural circular frequency of the ith mode, h_i is the damping constant of the ith mode, and $i > j$. The damping constants h_i and h_j are set to be 0.03. Eq. (2) is usually employed in the seismic response analysis of bridges in Japan.

The initial stress on the girder and towers is assumed to be that present under dead load conditions.

3. Possibility of cable loosening

3.1. Response characteristics for a one-direction earthquake

The bridge is analyzed seismically for ground motions in a single direction: longitudinal, lateral or vertical. The ground motions used for the analysis are the NS (north and south) and UD (up and down) components of the seismic wave observed at JR (Japan Railways) Takatori Station in the 1995 Great Hanshin Earthquake (Japan), which are shown in Fig. 6. The duration time is 50 s. The NS component is applied in the longitudinal or lateral direction and the UD component is applied in the vertical direction.

As a result, the cables loosen only when the earthquake motion is applied to the bridge in the longitudinal direction. Therefore, the following analysis is limited to the case in which the earthquake motion is applied to the bridge in the longitudinal direction. Moreover, the values under dead load condition are included in the seismic responses of the bridge, shown in the following figures.

3.1.1. Axial forces and axial displacements of cables

The loosening appears only in the bottom cables, i.e., cables C10 and C11 whose initial forces are relatively small, as shown in Table 1. Fig. 7 shows the deformation of the bridge at a specified moment in time.

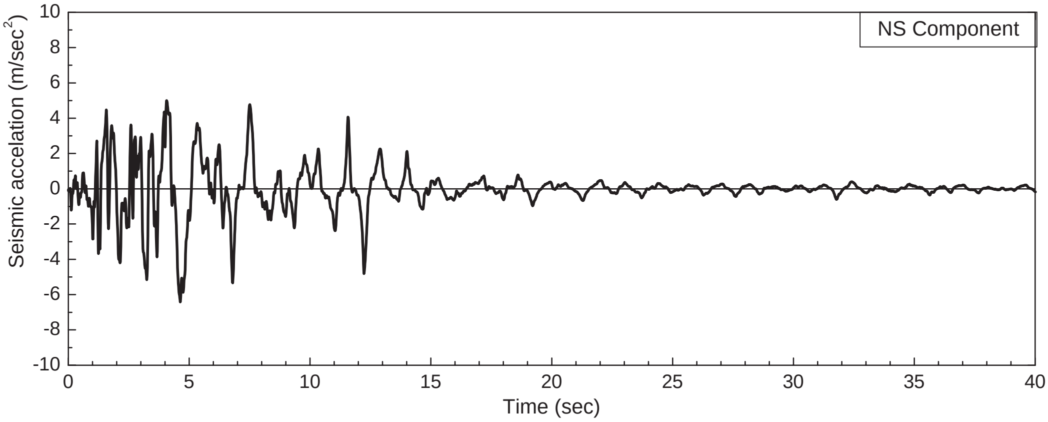
Fig. 8 shows the maximum axial force and axial displacement of the cables. Displacements of cables beyond the yield points are not found in the present analysis. The maximum axial forces of cables C10 and C11 in the non-linear cable model are greater than those in the linear cable model, but the axial displacements of the cables for the two models show only a small difference.

Fig. 9 shows the time histories of the axial forces and axial displacements of cables C10 and C11. The tensile force of cable C11 becomes large at the moment that cable C10 becomes loose (at about 5.2 s), as shown in Fig. 9(a) and (c). At the same time, the axial displacements in the non-linear cable model are greater than those in the linear cable model, as shown in Fig. 9(b) and (d).

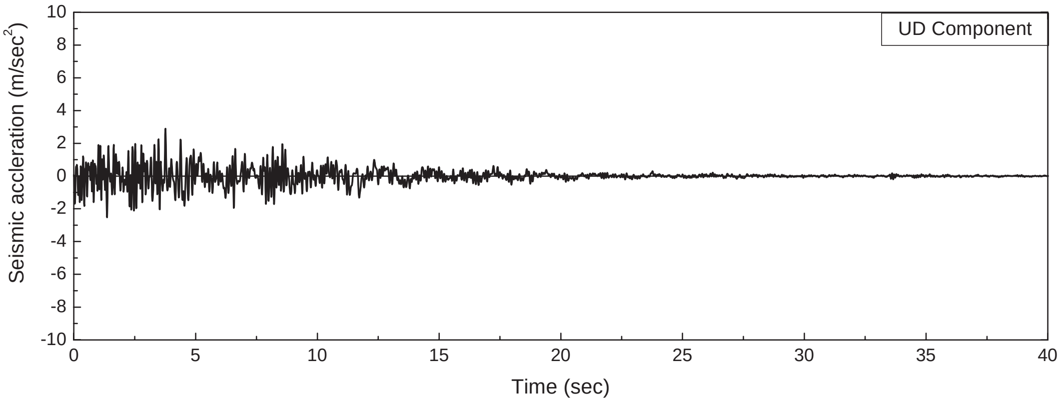
3.1.2. Responses of girder and towers

Fig. 10 shows the maximum bending moments of girder and the maximum axial force of tower. As can be seen in Fig. 10(a), the maximum bending moment at position B4 in the girder in the non-linear cable model is smaller than that in the linear cable model.

Fig. 11 shows the time history of the bending moment and the $M-\Phi$ curve at B4 in the girder. The bending moment at B4 (21 090 kN m) in the non-linear cable model is 20% smaller than that



(a) NS component



(b) UD component

Fig. 6. Seismic acceleration wave observed at JR Takatori Station.

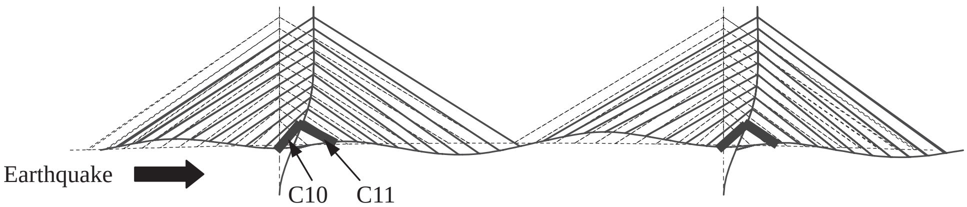
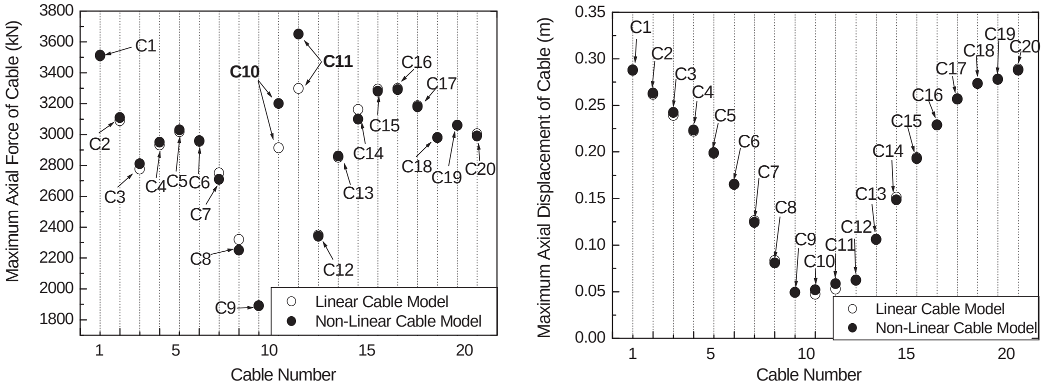


Fig. 7. Deformation of the bridge at time = 5.2 s.

(26 340 kN m) in the linear cable model since non-linear cable cannot take charge of compressive force.

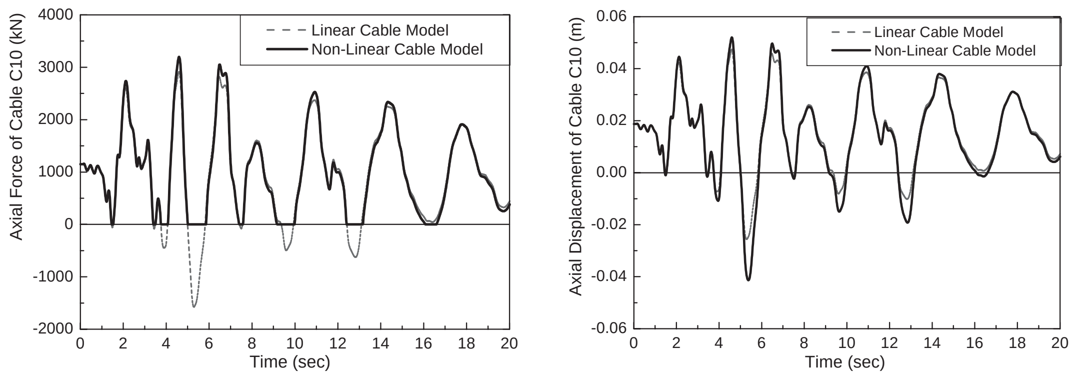
From Fig. 10(b), the maximum axial forces at the position T3 and T4 in the tower in the non-linear cable model are slightly greater than those in the linear cable model.



(a) Maximum axial forces

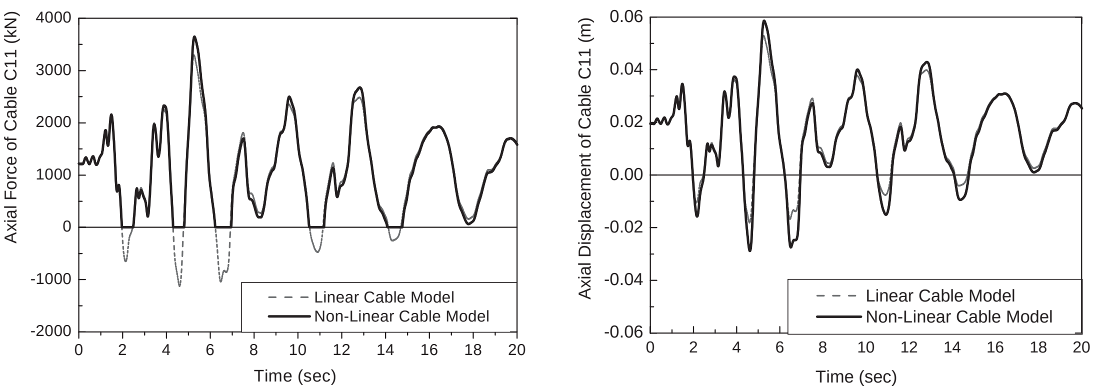
(b) Maximum axial displacements

Fig. 8. Maximum axial forces and displacements of cables.



(a) Axial force of cable C10

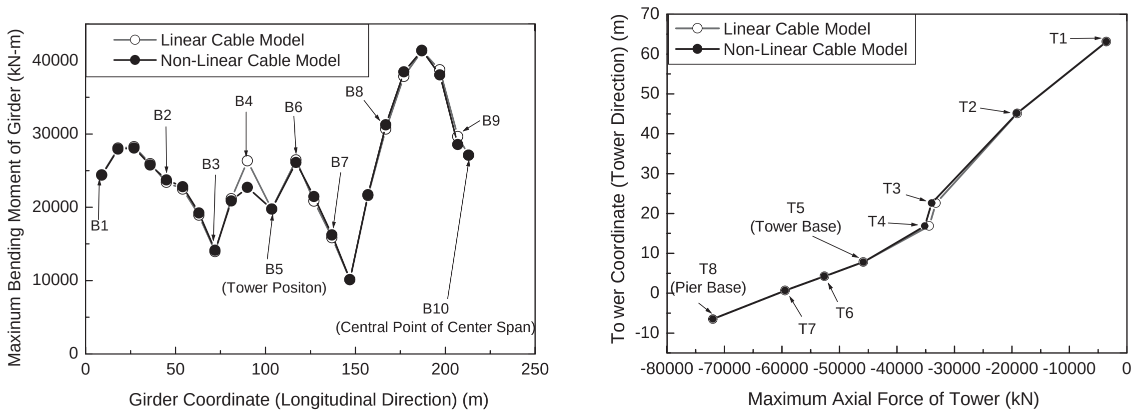
(b) Axial displacement of cable C10



(c) Axial force of cable C1I

(d) Axial displacement of cable C11

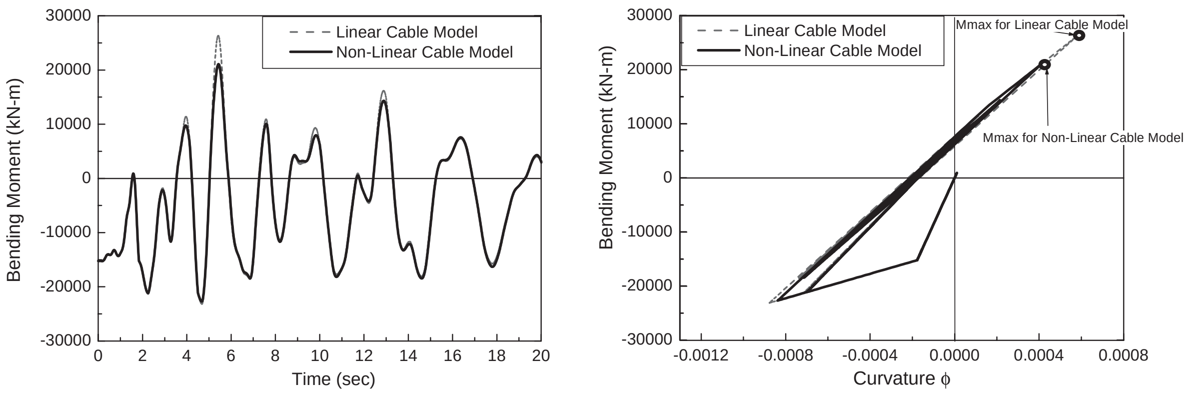
Fig. 9. Time histories of axial force and axial displacement of cables C10 and C11.



(a) Bending moments of girder

(b) Maximum axial forces of tower

Fig. 10. Maximum responses of girder and tower.



(a) Time history of bending moment

(b) M-Φ curve

Fig. 11. Bending moment at B4 in the girder.

Fig. 12 shows the time history of the axial force at T4 in the tower. The axial force at T4 ($-35\,140$ kN) in the non-linear cable model is also about 2% greater than that ($-34\,400$ kN) in the linear cable model.

From a similar discussion, it can be observed that the effect of the cable loosening on the displacement of towers and girder, bending moments of towers and axial forces of girder is small.

3.2. Response characteristics for in-plane earthquakes

The analysis shows that the difference in the responses of cables, girder and towers between the non-linear and the linear cable model is very small even if seismic motions are simultaneously applied in the longitudinal and vertical directions.

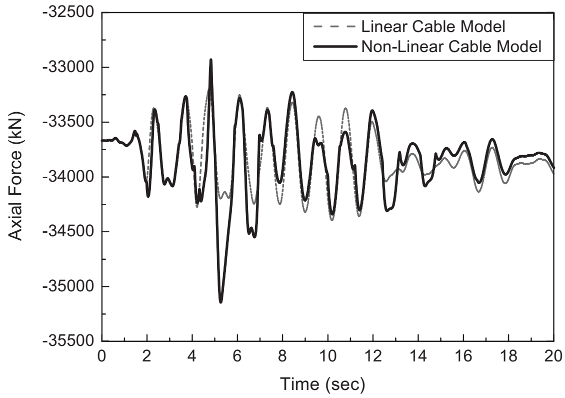


Fig. 12. Time history of axial force at T4 in the tower.

Table 2
Standard strong ground motions used in the analysis

ID	Solid condition	Observed station	DT (s)	PGA (cm/s^2)
T211	Group 1 (stiff)	1995 Kobe JMA (N–S)	30	−812.020
T212	Group 1 (stiff)	1995 Kobe JMA (E–W)	30	765.884
T213	Group 1 (stiff)	1995 Inagawa Bridge	30	780.046
T221	Group 2 (moderate)	1995 JR Takatori Stataion(N–S)	40	686.831
T222	Group 2 (moderate)	1995 JR Takatori Stataion (E–W)	40	−672.639
T223	Group 2 (moderate)	1995 Osaka GAS Fukiai Station (N–S)	40	736.334
T231	Group 3 (soft)	1995 Higashi Kobe Bridge	50	−591.034
T232	Group 3 (soft)	1995 Kobe Port Island (N–S)	50	−557.427
T233	Group 3 (soft)	1995 Kobe Port Island (E–W)	50	619.186

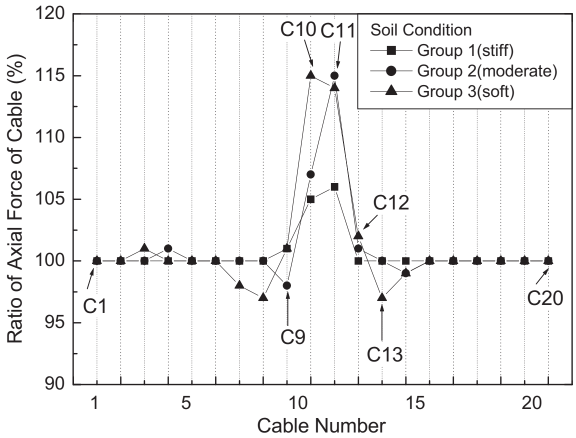
PGA: peak ground acceleration, DT: duration time, JMA: Japan Meteorological Agency.

4. Effect of cable loosening on the response

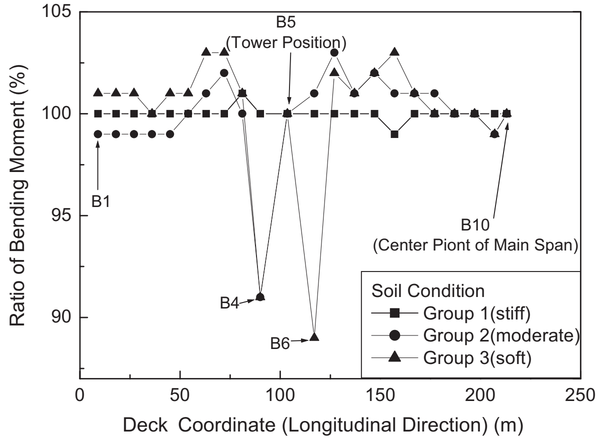
Generally, the characteristics of the ground motion have a great influence on the bridge response. Even if the ground motions have the same response spectrum, the bridge response can be changed. To solve this problem, Design Specifications for Highway Bridges 1996 [6] recommends that the mean value of maximum responses for at least three strong ground motions should be used to verify structural safety. In accordance with this code, this paper evaluates the maximum response using the mean value for three ground motions. The ground motions used in this analysis are shown in Table 2 [6].

Fig. 13 shows the mean value of the maximum responses of the cables, girder and towers. The vertical axis in Fig. 13(a) and (b) and the horizontal axis in Fig. 13(c) are the ratios of the responses between the non-linear and linear cable models.

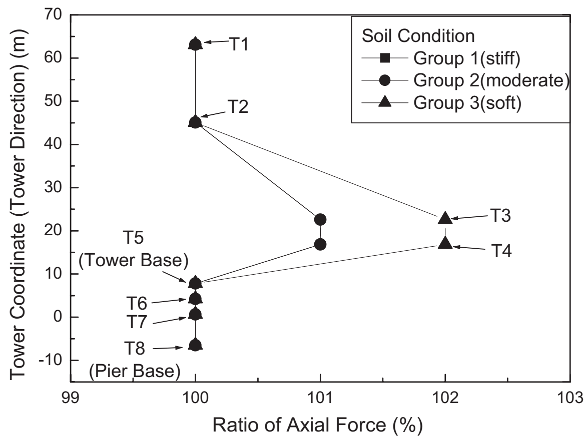
The tensile force of the cable on the opposite side of the tower may increase when there is the compressive force on the cable. The axial forces of cables C10 and C11 in the non-linear cable model are about 15% greater than those in the linear cable model, as shown in Fig. 13(a).



(a) Axial forces of cables



(b) Bending moments of girder



(c) Axial force of tower

Fig. 13. Mean values of maximum responses.

The larger effect of the cable loosening can be better seen in the part of the girder between B4 and B6 (Fig. 13(b)) and in the tower between T3 and T4 (Fig. 13(c)) than in other parts of the bridge. The influence of the non-linearity of the cables is most significant in soft soil condition (Group 3 [6]). However, the increase of the responses of the girder and towers is no more than 3% between those that consider the cable loosening and those that do not, while the decrease of the bending moment of girder is about 11%. Thus, the present analysis shows that the increase of the responses of the girder and the towers is not significant when the cable loosening is taken into account.

5. Concluding remarks

The cable loosening, the factor that is neglected in the conventional analyses, is considered in evaluating the response of a PC cable-stayed bridge. The results obtained in the present paper may be summarized as:

(1) The cable loosening appears only in the bottom cable of the multi-cable system. When the cable loosening is considered, both the axial force and axial displacement of the cables increase by about 15%, since the stiffness of the cables on the compressive side becomes zero. Yields of cables are not observed since the initial forces of the bottom cables are relatively small.
(2) The parts of the girder and towers surrounded by the bottom cables and the girder are affected by the cable loosening.
(3) The dynamic response of the girders and towers in a PC cable-stayed bridge is slightly increased when the cable loosening is considered.

From this analysis, the seismic responses of PC cable-stayed bridges are slightly increased except the bottom cables when cable loosening is considered. It is not necessary that cable loosening be considered to obtain seismic responses of girder and towers.

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